Kansas Citys, Missouri and Kansas
Flood Damage Reduction Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Interim Feasibility Report

Chapter A-13

STRUCTURAL ANALYSIS
ARGENTINE RAISE

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STRUCTURAL ANALYSIS – ARGENTINE RAISE

A-13.1 INTRODUCTION

The structural section of the Engineering Appendix for the Kansas Citys, Missouri and Kansas Flood Protection Project contains an evaluation of the existing floodwalls', gatewells', closure structures', and drainage structures' abilities to facilitate a raise in the levee's level of protection to meet Nominal 500-yr, Nominal 500-yr + 3ft, and Nominal 500-yr + 5ft flood events. The results of this phase of the study are used in the development of an economic benefit to cost ratio for each potential levee raise.

A-13.2 CRITERIA

A-13.2.1 Stability Requirements

Structural stability criterion used in the study of future conditions can be seen in Table A-13.1. It is based upon the draft EM 1110-2-2100 – *Stability Analysis of Concrete Structures*, dated 30 May 2001, with the exception of the extreme load condition. There is some concern with the extreme load condition categories as specified in EM 1110-2-2100. The Missouri River L-142 Design Criteria Issue Resolution Paper (2002) addressed these issues and put forth more stringent guidelines for recommended extreme load condition stability criteria. That criterion is used herein.

TABLE A-13.1 - Stability Criterion

Recommended Sliding Stability Factor of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Usual	10 yrs	2
Unusual	300 yrs	1.5
Extreme	Top of Protection	1.3*

Recommended Rotational Stability Percent of Base in Compression		
Load Condition Category	**Return Period**	**Percent of Base in Compression**
Usual	10 yrs	100%
Unusual	300 yrs	75%
Extreme	Top of Protection	25% *

Recommended Maximum Allowable Bearing Capacity % Increase in Allowable Bearing Capacity		
Load Condition Category	**Return Period**	**% Increase in Allowable Bearing Capacity**
Usual	10 yrs	0%
Unusual	300 yrs	15%
Extreme	Top of Protection	50%

Recommended Flotation Stability Factor of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Usual	10 yrs	1.3
Unusual	300 yrs	1.2
Extreme	Top of Protection	1.1

* Stability requirements increased from value in draft EM 1110-2-2100

A-13.2.2 Strength Requirements

For new structures designed with the Strength Design Method, loads are increased by multiplying service loads by appropriate load factors and nominal strengths are decreased by appropriate strength reduction factors. Load factors required by EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures* are a dead and live load factor of 1.7 and a hydraulic factor of 1.3. Combining these gives a total load factor of 2.2. The strength reduction factor for flexure, the typical controlling failure mechanism, is 0.90. Dividing the load factor by the strength reduction factor gives an overall factor of safety of about 2.45 for a new design.

Load and strength reduction factors were not used in the analysis of the existing structures. This implies that if an existing structure has a calculated Factor of Safety of less then 1.0, the structure has ceased to function as designed. When considering an allowable factor of safety for existing structures, several allowable reductions can be taken into account. EM 1110-2-2104 allows for a 25% reduction in load for short duration loads with a low probability of occurrence, which would apply to flood events with a return period of greater than 300 years. A "performance" factor can also be applied to take into account the previous behavior of the existing structure. Knowing that the existing structure has performed well under loading and not shown visible signs of distress, it is assumed a 15% reduction in factor of safety is acceptable. Combining the design factor with the frequency and performance factors produces an approximate 1.5 Factor of Safety for existing hydraulic structures in extreme loading conditions.

For structures subjected to earthen loads without extreme water loadings, such as unsubmerged box culverts and gatewells, the hydraulic load and extreme loading reduction factors would not apply. The resulting allowable factor of safety would include a 1.7 live load factor, a 0.90 flexural strength reduction factor, and a 15% factor of safety reduction for known performance of existing structures. Combining these load factors and strength reductions would result in a 1.6 allowable factor of safety for existing structures under normal (non-hydraulic) load conditions.

Risk and uncertainty analysis will be performed for any structures not meeting these minimum extreme and normal load condition Factors of Safety.

A-13.2.3 Uncertainty Analysis

For structures not meeting deterministic strength and stability criterion, a risk and uncertainty analysis was performed. A Taylor Series Method (TSM) of analysis is used in the calculation of structural risk and uncertainty. The TSM is appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). Examples of non-linear behavior for which the TSM should not be used include overturning stability analysis when the resultant is outside the kern of the base. Examples of degradation over time, which were not considered for the execution of this study, would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift.

Risk Calculation. a. For strength calculations, uncertainty is measured by applying a mean and standard deviation to the concrete and steel strengths. The selected mean and normal standard deviation are based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

b. For stability calculations, uncertainty is considered by applying a mean and standard deviation to the soil unit weight and shear strength, and is based on values provided by the geotechnical engineers working on the study.

c. Failure is defined as the capacity to demand ratio (factor of safety) less than 1.0, or in other words, when the demand (loads) exceed the capacity (structural or geotechnical).

Material Properties. a. For the screening portion of the Kansas Citys Flood Damage Reduction Feasibility Study, the following structural properties were used. The American Concrete Institute recommended the use of a 3,000 psi concrete strength around the 1940's and 1950's, the typical timeframe of construction for most of the levee structures in these feasibility studies. Limited design documentation and as-built drawings have been discovered that support the 3,000 psi original design strength assumption. For earlier concrete strengths, little information exists. It is currently assumed that 2000 psi concrete strengths are appropriate. If additional research information is discovered, this value will be updated.

b. Knowing the time period of construction (~1940's – 1950's) and based upon the Portland Cement Association's pamphlet *Engineered Concrete Structures*, 1997, an assumed reinforcing steel design yield strength, F_y, of 40 ksi is used for most computations, unless known or stated otherwise. This number has also been verified in the limited original design documents that have been found. For earlier structures (~1900's), the Concrete Reinforcing Steel Institute in *Engineering Data Report 48* suggests 33 ksi steel is typical.

c. Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength. For reinforced concrete structures Harr suggests a 14% standard deviation.

Concrete Strength Variation

1940's-1950's: $-\sigma = 3225$, $\mu = 3750$, $+\sigma = 4275$ (3000 psi min)
1900's-1920's: $-\sigma = 2150$, $\mu = 2500$, $+\sigma = 2850$ (2000 psi min)

Steel Strength Variation

1940's-1950's: $-\sigma = 43$, $\mu = 50$, $+\sigma = 57$ (40 ksi min)
1900's-1920's: $-\sigma = 35.5$, $\mu = 41.25$, $+\sigma = 47.0$ (33 ksi min)

A-13.3 ARGENTINE UNIT

A-13.3.1 Description of the Levee Unit - Structural Aspects

The Argentine Unit is located in Wyandotte County, Kansas on the right bank of the Kansas River between approximate Kansas River miles 10.1 and 4.75. The levee begins at Station 0+00 along the Barber Creek tieback and travels along the Santa Fe Railroad embankment to station 29+02 where a stop log gap spans six Santa Fe railroad lines. Timber stop logs are used to close the railroad openings. Moving east, or downstream, the levee continues to station 251+65 where a floodwall protecting the Argentine Boulevard Pump Station starts and then ends at Station 253+92. Earthen levee then continues to station 276+70 where the second of Argentine's two floodwalls extend east, adjacent to the Santa Fe Railroad tracks, to station 287+92. Both walls are inverted cantilever T-walls on spread footing foundations. A second stop log closure structure continues to station 288+57, crossing the same six lines of Santa Fe Railroad track and also using timber stop logs for closure.

Seventeen major gatewell closure structures are scattered along the length of the levee. Minor outlets with valve boxes associated with the pressure lines crossing the levee are addressed in the civil works utility portion of this study (see Civil Design Section of this Appendix). Four reinforced concrete box (RCB) culverts pass under the levee and service the pump plants at Stations 60+40 (Turner Station), 253+14 (Argentine Blvd.), 258+36 (Santa Fe Yards), and 273+41 (Strong Ave.).

A-13.3.2 Assumptions

Material properties could not be determined from existing documentation for a majority of the structures on the Argentine unit. As a result, estimated steel strengths, concrete strengths, and standard deviations were used for all strength analysis and risk computations for the structures on the Argentine unit (as noted in the previous section on uncertainty analysis).

Mean soil shear strengths and unit weights were assumed to be 28^{o} and 120 pcf respectively, based on the recommendations of the geotechnical engineers on the study team.

Before final design, reinforcing and concrete strengths shall be verified by testing.

A-13.3.3 Floodwalls

Using elevations from the design water surface profiles provided by the Hydrology and Hydraulics Branch in the *Hydraulic Analysis Existing Conditions Report* (2003), the required elevations for the nominal 500-year, nominal 500-year +3ft, and nominal 500yr +5ft levels of protection were established. Stability requirements were checked using the Army Corps of Engineers CASE project program CTWALL and were based on a floodwall raise consisting of a stem wall addition. The stem wall addition is made possible by pouring a new reinforced concrete extension doweled into the existing top of floodwall. The existing floodwalls' reinforcing was checked using unfactored loads and unreduced strengths to determine the adequacy of the existing reinforcement to sustain the increased loading. A summary of results is displayed in Table A-13.2.

TABLE A-13.2 - Nominal 500-yr Floodwall Performance

Floodwalls with Nominal 500-yr Wall Raise (2.4 ft) Extreme Condition (Water at Top of Wall)			
Criteria	**Required**	**Floodwall 251+56** (+2.4 ft Wall Raise)	**Floodwall 276+70** (+2.4 ft Wall Raise)
Sliding Stability	> 1.3 Factor of Safety	N/A*	0.89
Rotational Stability	> 25% Base in Compression	N/A*	0%
Bearing Pressure	< 150% Increase in Allowable Bearing Pressure	N/A*	N/A
Strength	> 1.5 Factor of Safety for Existing Structure	N/A*	0.63**

* Floodwall to be abandoned / removed with Argentine Pump Station replacement.
**Factor of Safety of wall with heel addition to ensure sliding and rotational stability, and without buttresses or counterforts.

It was determined that, with the nominal 500-year wall raise, the floodwall from Station 276+70 to 287+92 would not meet stability criteria and modifications would be necessary. The easiest alternative was the addition of backfill behind the existing wall. Calculations using CTWALL showed that five feet of fill would be required to make the wall stable. Due to the close proximity of the railroad, such a raise was deemed impractical. Modifying the foundation by extending the toe, expanding the heel, and a combination of expanding the heel and extending the toe, were also analyzed using CTWALL. From this analysis, a heel addition was decided to be the most practical.

Strength analysis of the modified floodwall at Station 276+70 revealed that the wall is under-reinforced and is not capable of carrying the additional loading associated with the nominal 500-year flood elevation wall raise (2.4ft). Buttresses and counterforts would be needed to strengthen the wall.

Costs comparison takeoffs were performed in order to determine if expanding the heel of the foundation, adding buttress and counterforts, and adding a 2.4 ft wall extension would be more cost effective then removing the existing wall and replacing with a new floodwall. As a result of the extensive labor requirements necessary to modify the existing wall, the cost estimate revealed that it would be more cost effective to remove and replace the existing wall than to attempt to modify the wall. It should be noted that a temporary levee to maintain a 100-year level of flood protection was not necessary for this cost estimate. Such protection is not required because the landside area is higher then the 100-year event. To reestablish the existing level of protection in the occurrence of a flood event, a temporary earthen levee can be constructed to minimize flooding. For additional information see Chapter A-17, Construction Procedures and Water Control Plan.

Current plans call for the removal of portions of the floodwall surrounding the Argentine Pump Station located at station 251+56 in order to facilitate the replacement of the existing Argentine Station Pump plant. The remaining floodwalls will be removed to approximately 2 feet below the ground surface and buried in place. For a more detailed description of this procedure, see the Pump Station Analysis chapter of this report.

A-13.3.4 Stop Log Gaps

Using the same stability criteria as required for floodwalls and described in Table A-13.1, the two stop log gaps at Stations 29+02 and 288+57 were also reviewed for strength and stability requirements. The results are shown in Table A-13.3 below.

TABLE A-13.3 – Stop Log Gap Performance

Stop Log Gaps with Nominal 500-yr Wall Raise Extreme Condition (Water at Top of Wall)			
Criteria	Required	**Stop Log 29+02** (+1 ft Wall Raise)	**Stop Log 288+57** (+1.82 ft Wall Raise)
Sliding Stability	> 1.3 Factor of Safety	2.6	1.28
Rotational Stability	> 25% Base in Compression	71%	61%
Bearing Pressure	< 150% Increase in Allowable Bearing Pressure	60%	>> 150%
Strength	> 1.5 Factor of Safety for Existing Structure	1.57	0.75*

*Factor of Safety of stop log gap with foundation modification to reduce bearing pressure and without buttresses or counterforts.

Modifications of the foundation are required for the stop log gap at Station 288+57. Using CTWALL, it was determined that a heel extension would be sufficient in reducing bearing pressures to allowable levels. In an effort to reduce the extensive labor expense associated with dowelling the heel extension into the existing stop log gap foundation, it was decided to fill the area between the riverside sheet pile cutoff wall and stem with concrete to achieve an equivalent stability. Table A-13.4 summarizes the resulting stability Factors of Safety for the modified stop log gap at Station 288+57.

TABLE A-13.4 - Factor of Safety for Modified Stop Log Gap 288+57

Stability of Modified Stop Log Gap with 500-yr Wall Raise Extreme Condition (Water at Top of Gap)		
Stability Criteria	**Required**	**Stop Log 288+57** (1.82 ft Raise)
Sliding Stability	> 1.3 Factor of Safety	1.67
Rotational Stability	> 25% Base in Compression	64%
Bearing Pressure	< 150% Increase in Allowable Bearing Pressure	80%

It is the recommendation of this study that, for the nominal 500-yr levee raise, the stop log gap at Station 29+02 will require a 1 foot wall raise. The stop log at 288+57 will require a 1.82 ft wall raise, landside buttresses, and a riverside foundation modification. For both the nominal 500-yr +3ft and nominal 500-yr +5ft levels of protection, new replacement stop log gap closure structures shall be required at Stations 29+02 and 288+57.

A-13.3.5 Gatewells and Outlets

The seventeen gatewells along the Argentine unit were analyzed for uplift and strength requirements to determine if the gatewells can be modified for a levee raise. Uplift Factors of Safety were calculated using Draft EM 1110-2-2100 and strength factors of safety using unfactored loads and unreduced strengths. Results are summarized in Table A-13.5. All values are based on the nominal 500-yr + 5ft flood event (worst case).

Two outlets at Stations 13+75 and 210+73, without positive riverside closure, were inspected on an April 27, 2004 site visit to determine if additional gatewells would be required at these locations to supply positive closure. The examination revealed the 15" CMP at Station 13+75 to be unnecessary and the 16" CIP at Station 210+73 to be abandoned and filled with soil and debris. It is recommended that both be properly abandoned with non-shrink grout fill in accordance with Army Corps of Engineers criteria. These findings are summarized in Table A-13.5.

As a result of this investigation, it is recommended that the gatewell at Station 284+35 should be replaced due to the fact that the existing floodwall it is constructed in must be replaced. For all other gatewells, a wall extension only is required.

For the gatewell raises, the existing top slab of the gatewell will be removed, wall extensions poured, and then a new top slab poured. Such a procedure will allow a greater ease of access into the gatewell and prevent possible entrance problems associated with navigating through multiple floor slab openings.

TABLE A-13.5 – Gatewell & Outlet Summary

Station	Exterior Dimensions (ft)	Pipe	Uplift Factor of Safety (> 1.1 FoS Req)	Strength Factor of Safety (> 1.5 FoS Req)	Comments and Recommendations
13+75	No Gatewell Present	15" CMP	N/A	N/A	Site inspection on 04/27/04 showed 12" CMP draining small area with buried inlet. Recommendation made to properly abandon pipe per Corps Guidelines.
35+10	6 x 6.5	36 " RCP	1.4	1.9	Gatewell Extension.
60+40	11.5 x 14	2 – 5' x 8' RCB	N/A	N/A	Gatewell located on landside of levee crest. Recommend gatewell extension only.
97+70	5.5 x 5.5	8" CIP	1.5	2.4	Gatewell extension with pipe up and over levee.
131+37	6.5 x 7.5	12" SP	N/A	N/A	Gatewell on landside in Bulk Mail Center pump plant. Recommend gate well extension with pressure pipe up and over levee.
131+50	8.5 x 9.25	48" RCP	1.3	1.7	Gatewell Extension.
145+00	6 x 6.5	36" RCP	1.4	2.1	Gatewell Extension.
190+00	7.83 x 10.33	60" RCP	1.6	2.0	Gatewell Extension.
210+73	No Gatewell Present	16" CIP	N/A	N/A	Site inspection on 04/27/04 showed improperly abandoned 16" CIP filled with soil and debris. Recommendation made to properly abandon pipe per Corps Guidelines.
218+17	7 x 7.33	36" RCP	1.3	2.1	Gatewell Extension.
247+32	5.5 x 5.5	24" CIP	1.5	2.6	Gatewell Extension.
253+14	13 x 14	9' x 9.5' RCB	1.2	1.9	Gatewell Extension.
258+36	6 x 12.5	4' x 5.5' RCB	N/A	N/A	Gatewell on landside in Bulk Mail Center pump plant. Recommend gatewell extension only.
273+41	10 x 12	7' x 7' RCB	1.3	2.8	Gatewell Extension.
280+48	6 x 6.5	36" RCP	1.4	1.9	Gatewell Extension.
284+35	6.75 x 14	10' x 10' Spillway	N/A	N/A	Gatewell for Ruby Avenue 10' x 10' spillway. Existing structure extensively constructed with existing floodwall in that section that is to be replaced. Recommended to replace gatewell
288+10	6 x 6	6" CIP	1.5	2.9	Gatewell Extension.

A-13.3.6 I-Wall

The close proximity of railroads, buildings, and possible hazardous or toxic waste contamination to the toe of the levee along reaches of the Argentine Unit does not allow for the placement of an earthen levee raise, as determined by the geotechnical members of the project team. A sheet pile with concrete capped I-Wall along the levee crest will be used in the reaches with limiting landside constraints. A typical I-wall cross-section is shown in Exhibit A-13.1. Table A-13.6 lists the regions where I-wall will be required and descriptive dimensions. Preliminary computations in the Army Corps of Engineers CASE project Program CWALSHT verified that the maximum sheet pile depth of fifteen feet for the nominal 500-yr + 5ft will meet stability criteria.

EXHIBIT A-13.1 - Typical I-Wall Cross-Section

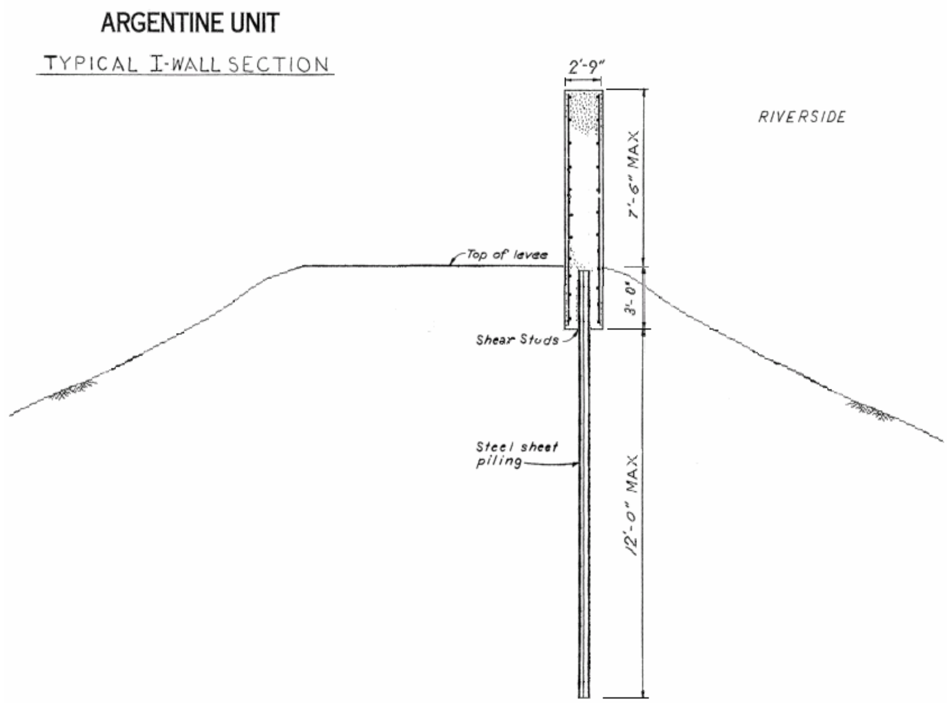


TABLE A-13.6 - I-Wall Locations

I-Wall					
Nominal 500-year					
Beginning Station	Ending Station	Length (ft)	Total Wall Height (ft)	Exposed Height (ft)	PZ35 Sheet Pile (ft)
27+50	28+30	80	4	1	0
Nominal 500-year + 3 ft					
Beginning Station	Ending Station	Length (ft)	Total Wall Height (ft)	Exposed Height (ft)	PZ35 Sheet Pile (ft)
-2+00	28+30	3030	6.5	3.5	10
61+00	118+00	5700	6.5	3.5	10
245+00	251+65	665	8	5	11
253+92	276+70	2268	8.5	5.5	11
289+09	289+40	31	9	6	11
Nominal 500-year + 5 ft					
Beginning Station	Ending Station	Length (ft)	Total Wall Height (ft)	Exposed Height (ft)	PZ35 Sheet Pile (ft)
-3+00	28+30	3130	8.5	5.5	11
61+00	118+00	5700	8.5	5.5	11
245+00	251+65	665	10	7	15
253+92	276+70	2268	10.5	7.5	15
289+09	289+40	31	11	8	15

A-13.3.7 Reinforced Concrete Box Culverts

In the 1970s, the Argentine unit was raised approximately 5 feet and the four box culverts were loaded with additional fill. No modifications were made to the boxes to accommodate the additional fill. Analysis of the existing boxes shows the boxes at Stations 60+40, 253+14, and 258+36 to be adequate, but the box at Station 273+41 to be deficient. Table A-13.7 shows a summary of these findings. For more detailed information on the analysis and the purposed fix for the deficient box, see the Existing Conditions Addendum to Argentine Analysis section of the Existing Conditions chapter within this appendix.

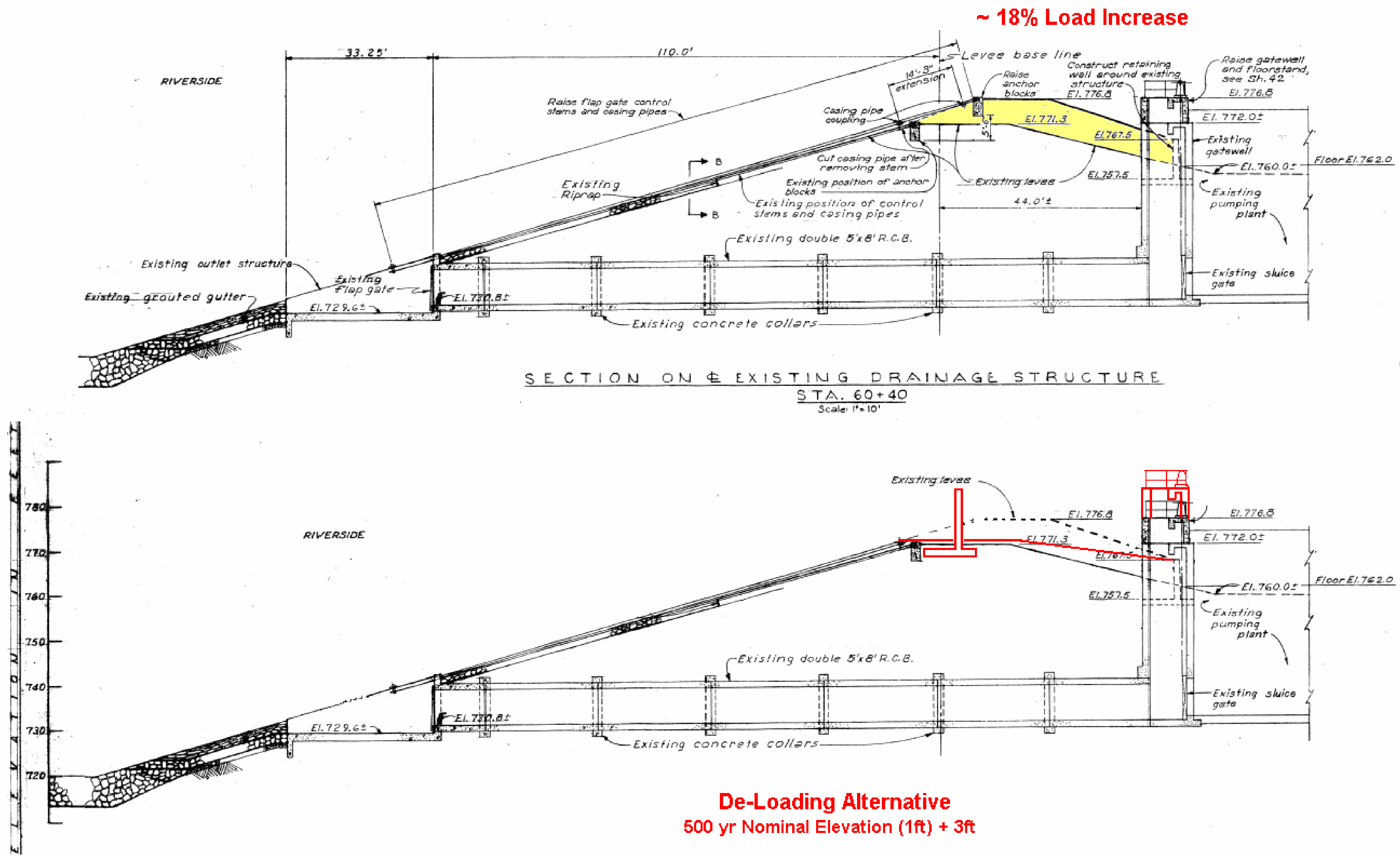
TABLE A-13.7 - RCB Summary

Name / Station	Description	Factor of Safety* (Existing Conditions)	Comments	Recommendations (Future Conditions)
Turner Station 60+40	Twin 5' x 8' RCB	2.21	Field observations on 07-April-04 collaborate the overall functionality of the RCB.	A De-Loading approach is recommended. (See illustration)
Argentine 253+14	9.5' x 9' RCB	2.32	Field observations on 07-April-04 collaborate the overall functionality of RCB.	To be abandoned / removed with Argentine Pump Station Replacement
Santa Fe Yards 258+36	4' x 5' RCB	5.00	No field observations attempted due to high Factor of Safety.	A levee raise may be acceptable. A detailed inspection will be required at time of Plans and Specs to insure adequacy of structure.
Strong Ave. 273+41	7' x 7' RCB	1.07 - 0.75	Limited observations on 07-April-04 due to sewer diversion.	A steel pipe will be inserted into the 7x7 RCB and grout fill used between the pipe and RCB.

*For further information see Existing Conditions Addendum to Argentine Analysis section of the Existing Conditions chapter.

In order to minimize modifications and costs for a levee raise, a "De-Loading" alternative was investigated. De-Loading is intended to provide additional levee protection without adding load to the boxes. By removing earthen levee and replacing it with taller reinforced cantilever T-floodwalls, the vertical loading will remain the same while achieving an increase in protection. Exhibit A-13.2 illustrates a typical cross section for such a De-Loading approach.

EXHIBIT A-13.2 - De-Loading at Station 60+40



A-13.4 REFERENCES

1. US Army Corps of Engineers (1999), *Reconnaissance Report – Kansas Citys, Missouri and Kansas Flood Damage Reduction Project*, Kansas City District.
2. US Army Corps of Engineers (2003), *Kansas Citys, Missouri and Kansas Flood Damage Reduction Project Feasibility Study. Appendix B - Missouri and Kansas River Hydraulic Analysis Existing Conditions Report*, Kansas City District.